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SHEAR BEHAVIOUR OF COLD-FORMED STAINLESS STEEL LIPPED CHANNELS WITH REDUCED SUPPORT RESTRAINTS

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Abstract: Lipped channel beams (LCBs) are commonly used in buildings for load-bearing components such as floor joists and roof purlins. The typical practice is to use one-sided web side plates (WSPs) to attach beams from their webs to the supports at the connections, through bolts. In such realistic conditions, it is not practical to use WSPs over the full height of the webs, thus only a part of the web height is restrained at the supports. This controls the mobilising of diagonal tension field in the web and also provides less restraint to the lateral movement of the web. Therefore, realistic support conditions affect the shear capacity due to the lack of restraint of the web at the supports. On the other hand, the current shear design rules are based on ideal support conditions which do not represent the true scenario. Therefore, it is critical to investigate the effect of reduced support restraints on the shear capacity since it has been given less attention in the literature. This paper presents the effect of reduced support restraints on the shear capacity of stainless steel lipped channel beams. Finite Element (FE) models were developed to study the effect with regard to various influential parameters. From the FE results, it was found that the shorter the WSP - the higher the shear capacity reduction, where about 50 % shear capacity reduction was observed for 60 % reduction in WSP height. Furthermore, it was concluded that compact sections exhibit more significant capacity reduction than slender sections when reducing the WSP height. Therefore, a reduction factor was introduced to the current direct strength method (DSM) shear design rules considering the effect of reduced support restraints on the shear capacity.

Keywords: Reduced support restraints; Stainless steel; Lipped channel beams; Shear design rules; Direct strength method

1. Introduction

Lipped channel beams (LCBs) are commonly used as floor joists and roof purlins in steel structures. Figure 1 shows application of lipped channel sections as floor joists. They act as intermediate members to transfer loads from floors to columns. Different forms of simple connection types are used between these members which include double and single angle web cleats, header plates and fin plates. Due to the single symmetric nature of LCBs single angle web cleat connections, are the most common among lipped channel sections. At the connections these plates are bolted or screwed to the web. However, in the realistic conditions restraint provided by these connections is limited only to a part of the web height. Sufficient level of restraint provided by the supports is important in mobilising diagonal tension fields in spans, also known as tension field action (TFA). TFA for a lipped channel section is shown in Figure 2. Mobilising of TFA is essential for a section to achieve its shear capacity. The effect of support conditions on TFA of cold-formed lipped channel sections subject to shear has been investigated by Pham and Hancock (2012) while in a more recent study by Pham et al. (2017), the effect of flange restraints on TFA has also been discussed. Moreover, Keerthan et al. (2015) has investigated the effect of reduced support conditions on the shear behaviour of cold-formed hollow flange channel sections via experimental studies. In addition to these, shear behaviour of cold-formed lipped channel sections has been studied in a number of researches by Keerthan and Mahendran (2013a; 2015a). Moreover, Keerthan and Mahendran (2012) and Mahendran and Keerthan (2013) has investigated the shear behaviour of cold-formed LiteSteel beams. Further, studies have been conducted on the web crippling behaviour of cold-formed lipped channel sections by Sundararajah et al. (2017a; 2017b). However, limited attention has been given to stainless steel lipped channel sections.

Figure 1: Lipped channel beams as floor joists (Pham et al., 2017)

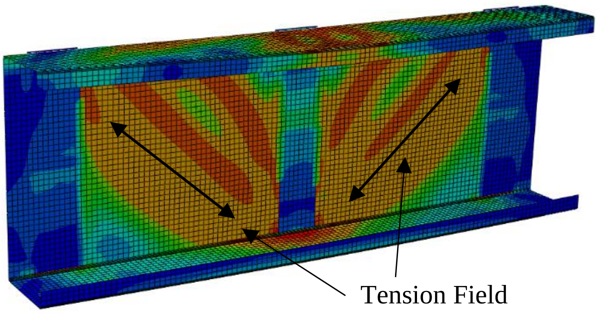


Figure 2: Tension field action

As a result of TFA, post-buckling strength can be observed particularly in slender sections subject to shear. This post-buckling strength available in cold-formed channel sections has been investigated in a number of studies conducted by Keerthan and Mahendran (2013b; 2015b). Increased support restraints improve the post buckling strength, especially in sections subject to shear (Pham et al., 2017). Reduced restraints at supports in realistic conditions affect the mobilising of TFA, thus reducing post-buckling strength and shear capacity of sections. On the other hand, current shear design provisions do not account for these capacity reductions, therefore relevant capacity reduction factors to those design rules should be introduced. Consequently, this paper presents the detailed finite element (FE) study conducted to investigate the effect of the reduced support restraints on the shear behaviour of cold-formed stainless steel lipped channel sections. FE results were utilised in determining the capacity reduction factors for current DSM shear design rules and all the details are also presented herein.

2. DSM Shear Design Rules

The direct strength method (DSM) was first introduced as an alternative to the conventional effective width method. It was adopted in both North American (AISI S100, 2007) and Australian (AS/NZS 4600, 2005) design standards for cold-formed steel. These DSM shear design rules with and without TFA are summarised below.

2.1 Without Tension Field Action

For section webs without holes and without any stiffeners, shear capacity (V_v) of the section without TFA is given by Eqs. (1)-(3). These equations do not account for the available post-buckling strength as well.

$$V_v = V_y \text{ for } \lambda \leq 0.815 \quad (1)$$

$$V_v = 0.815\sqrt{V_{cr}V_y} \text{ for } 0.815 < \lambda \leq 1.227 \quad (2)$$

$$V_v = V_{cr} \text{ for } \lambda > 1.227 \quad (3)$$

where V_y is the shear yield capacity and V_{cr} is the elastic shear buckling capacity. V_y and V_{cr} are defined by Eqs. (4) and (5), respectively. λ is the section slenderness and is expressed in Eq. (6).

$$V_y = 0.6\, f_y d_1 t_w \quad (4)$$

$$V_{cr} = \frac{k\pi^2 E t_w{}^3}{12\,(1-\upsilon^2) d_1} \quad (5)$$

$$\lambda = \sqrt{\frac{V_y}{V_{cr}}} \quad (6)$$

where f_y is the web yield stress, d_1 is the flat depth of the web, t_w is the web thickness, E is the Young's modulus and υ is the Poisson's ratio. Here, k is the shear buckling coefficient of the section. Keerthan and Mahendran (2015a; 2015b) proposed a set of equations to calculate the shear buckling coefficient (k) of LCBs considering the additional fixity available at the web-flange juncture of LCBs.

2.2 With Tension Field Action

Section shear capacity including TFA is given by Eqs. (7) and (8). These equations are in the form of the DSM section moment capacity equations and include post-buckling strength of the sections as well.

$$V_v = V_y \text{ for } \lambda \leq 0.815 \quad (7)$$

$$V_v = \left[1 - 0.15\left(\frac{V_{cr}}{V_y}\right)^{0.4}\right]\left(\frac{V_{cr}}{V_y}\right)^{0.4} V_y \text{ for } \lambda > 0.815 \quad (8)$$

2.3 Design curve for stainless steel sections

Based on the experimental and numerical studies current DSM shear design provisions were modified by Dissanayake et al. (2019a) to predict the shear capacity of stainless steel LCBs. Those proposals include both post-buckling strength of slender sections and inelastic reserve capacity of compact sections. Eqs. (9)-(11) give this modified rules for stainless steel.

$$V_v = 2V_y \text{ for } \lambda \leq 0.122 \quad (9)$$

$$V_v = 0.795\left(\frac{V_{cr}}{V_y}\right)^{0.439} V_y \text{ for } 0.122 < \lambda \leq 0.592 \tag{10}$$

$$V_v = \left[1 - 0.213\left(\frac{V_{cr}}{V_y}\right)^{0.35}\right]\left(\frac{V_{cr}}{V_y}\right)^{0.35} V_y \text{ for } \lambda > 0.592 \tag{11}$$

3. Numerical Modelling

3.1 Finite element model development

Details of finite element (FE) modelling conducted to investigate the effect of shear behaviour of stainless steel lipped channel sections with different support heights is presented in this section. Developed FE models were first validated and then utilised in a detailed parametric study. Abaqus software package was used in this study for the FE modelling. FE modelling was based on the experiments carried out to study the shear behaviour of stainless steel LCBs. In the experiments, two LCBs were attached back-to-back using three T-stiffeners and full-height web side plates (WSPs). LCBs were simply supported from the two ends and loaded at the mid-span using displacement control. Equal angle straps were provided to the both top and bottom flanges at the supports and at the loading point to avoid distortional buckling. More details on the experimental results can be found in Dissanayake et al. (2019a; 2019b) and Fareed et al. (2019).

In the FE modelling procedure, single LCBs were modelled considering the symmetry of the experimental setup. A pin and a roller support conditions were maintained at the tow ends. Rotational degree of freedom around the longitudinal axis of the beam was restrained to avoid any torsional effects. Support conditions and loading were applied to the single WSPs where a single node of each WSP was restrained at the shear centre of the section. In the validation study, the full web height was restrained using WSPs to simulate experimental conditions while in the parametric study further FE models were developed restraining only a part of the web height. WSPs with reduced heights were attached symmetrically along the web height. A bolted connection between the section web and the WSPs was modelled using Tie constraints available in Abaqus software. Similar procedure has previously been followed for FE modelling of cold-formed sections by Keerthan and Mahendran (2011) and Keerthan et al. (2014).

S4R shell elements were employed to define the sections. Following a sensitivity study, it was concluded that a 5 mm × 5 mm mesh offers the convergence with a reasonable accuracy. However, for the corner regions, a relatively finer mesh of 1 mm × 5 mm was assigned. Figure 3 illustrates the boundary conditions and FE mesh used.

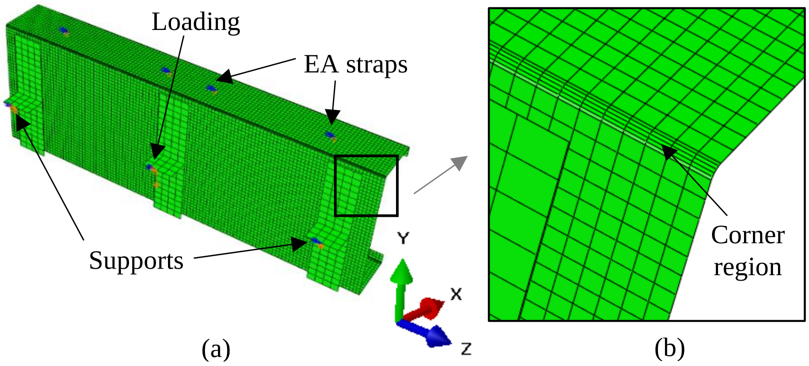


Figure 3: (a) Boundary conditions and (b) FE mesh used in the modelling

Modified two-stage Ramberg-Osgood material model proposed by Arrayago et al. (2015) was used to represent the non-linear stress-strain behaviour of stainless steel. Corner strength enhancement was incorporated to the FE models as defined in Dissanayake et al. (2019c). In order to introduce the effect of local geometric imperfections to the non-linear FE models an elastic buckling analysis was initially performed and critical buckling mode shapes were identified. Then those were superimposed to the non-linear FE models using a suitable imperfection amplitude. Modified Dawson and Walker model proposed by Gardner and Nethercot (2004) was used in this study for the calculation of imperfection amplitude (ω_0) and is given by Eq. (12). Thereafter, a modified Riks analysis was performed on FE models.

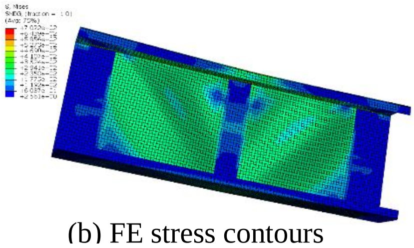
$$\omega_0 = 0.023\left(\frac{\sigma_{0.2}}{\sigma_{cr}}\right)t_w \tag{12}$$

where $\sigma_{0.2}$ is the 0.2 % proof stress and σ_{cr} is the critical elastic buckling stress of the slenderest element of the section.

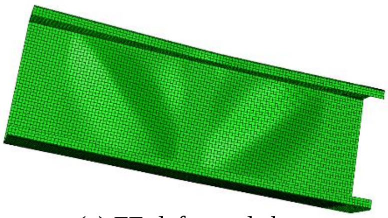
3.2 Validation

Table 1 summarises the experimental ultimate shear capacities of 8 LCBs with FE predictions for full web height restrained support condition. From Table 1 it can be seen that mean and coefficient of variance (COV) of the experimental to FE shear capacity ratio are 1.02 and 0.073, respectively. Therefore, the elaborated FE models predict ultimate shear capacity of LCBs with reasonably good accuracy. Furthermore, Figure 4 compares the experimental and FE shear failure modes for LCB 200×75×15×1.2 section. The FE model is able to capture diagonal shear failure of the web in both spans accurately.

(a) Experiment



(b) FE stress contours



(c) FE deformed shape

Figure 4: Shear failure mode of LCB 200×75×15×1.2 section

Table 1: Comparison of experimental and FE shear capacities

LCB section	**$V_{Exp.}$ (kN)**	**V_{FE} (kN)**	**$V_{Exp.}/V_{FE}$**
LCB 100×50×15×1.2	18.49	16.86	1.10
LCB 100×50×15×1.5	24.44	23.90	1.02
LCB 100×50×15×2.0	36.00	32.72	1.10
LCB 150×65×15×1.2	21.60	20.09	1.08
LCB 150×65×15×1.5	26.26	28.40	0.92
LCB 150×65×15×2.0	43.55	42.60	1.02
LCB 200×75×15×1.2	22.98	22.97	1.00
LCB 200×75×15×2.0	47.05	52.11	0.90
Mean			1.02
COV			0.073

3.3 Parametric study

In order to study the level of restraints provided by the support on shear TFA of stainless steel LCBs 80 FE models were developed following the validation. Four WSP heights (h_{WSP}) were considered in this study and the details of FE results are summarised in Table 2.

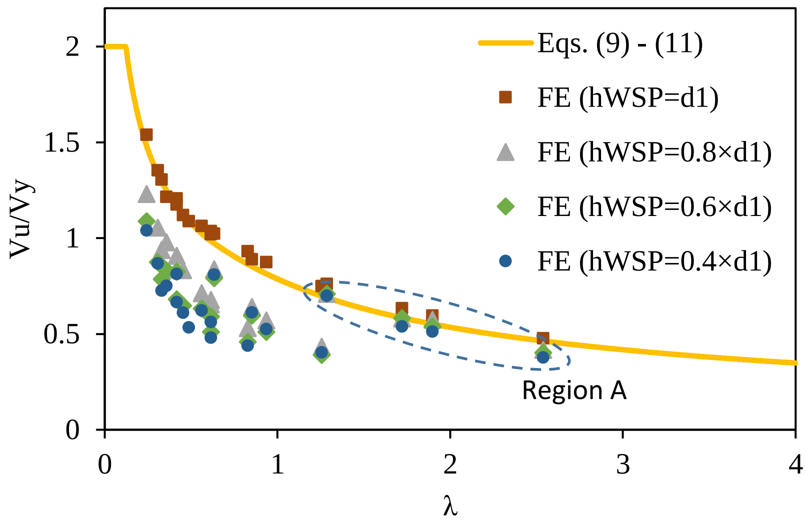


Figure 5: FE shear capacities of LCBs with reduced WSP heights and DSM shear design curve

Figure 5 plots the ultimate shear capacities of 80 FE models against the section slenderness (λ). It also includes the DSM shear design curve given by Eqs. (9)-(11). By analysing the results, it can be seen that the shorter the WSP – the higher the shear capacity reduction, where about 50 % shear capacity reduction can be observed for 60 % reduction in WSP height. When reducing the WSP height restraint provided by the support also reduces. Therefore, it can be concluded that insufficient level of restraint provided to fully mobilise TFA in webs causes this shear capacity reduction. Also, this capacity reduction is found to be higher in compact sections for the most cases considered.

Table 2: Summary of parametric study results

Section	**Shear capacity (kN)**			
	$h_{WSP}=d_1$	$h_{WSP}=0.8\times d_1$	$h_{WSP}=0.6\times d_1$	$h_{WSP}=0.4\times d_1$
Grade 1.4301				
LCB 150×65×15×1	15.6	14.5	14.5	14.3
LCB 150×65×15×2	41.2	33.8	31.9	32.6
LCB 150×65×15×3	72.0	54.2	49.2	48.5

LCB 150×65×15×4	106.2	82.7	68.6	68.0
LCB 150×65×15×5	148.7	118.9	105.2	100.5
LCB 200×75×15×1	17.3	15.9	15.9	14.8
LCB 200×75×15×2	48.1	34.7	32.3	33.1
LCB 200×75×15×3	85.4	57.3	50.8	50.1
LCB 200×75×15×4	124.6	88.7	72.2	70.6
LCB 200×75×15×5	171.2	123.1	103.2	95.1
Grade 1.4462				
LCB 150×65×15×1	26.5	25.5	24.0	22.8
LCB 150×65×15×2	76.6	50.0	44.8	46.1
LCB 150×65×15×3	134.3	87.9	76.6	73.0
LCB 150×65×15×4	190.9	142.1	110.5	104.2
LCB 150×65×15×5	255.3	205.4	174.6	157.7
LCB 200×75×15×1	28.4	24.8	23.9	22.5
LCB 200×75×15×2	88.2	51.1	46.0	47.4
LCB 200×75×15×3	162.7	92.7	80.1	76.7
LCB 200×75×15×4	235.1	150.9	117.8	110.6
LCB 200×75×15×5	310.3	215.5	174.4	152.5

4. Reduction factor for DSM Shear Design Rules

Shear capacities of stainless steel LCBs with reduced WSP heights lie well below the current DSM shear design curve for stainless steel according to Figure 5. Therefore, to fit these points closer to the current design curve, suitable reduction factor (RF) was proposed following a regression analysis. When proposing this reduction factor, FE data in the region A of Figure 5 were not considered. Then the proposed reduction factor was introduced together with the current DSM shear design curve to predict shear capacity of stainless steel LCBs with reduced support restraints in Eq. (13). The distribution of FE shear capacities after applying the proposed reduction factor is compared with the current DSM design curve and shown in Figure 6. Actual-to-predicted shear capacity reduction ratio has a mean and COV of 1.00 and 0.097, respectively, therefore highlight the accuracy of the developed expression.

$$\mathrm{RF} = 0.195 + 0.479\left[\frac{\mathrm{h_{WSP}}}{\mathrm{d_1}}\right]^{6.424} + 0.117\left[\frac{\mathrm{f_u}}{\mathrm{f_y}}\right]^{0.852} + 0.175[\lambda]^{-0.183} \text{ for } \frac{\mathrm{h_{WSP}}}{\mathrm{d_1}} < 1 \tag{13}$$

where f_u is the ultimate tensile stress of the material.

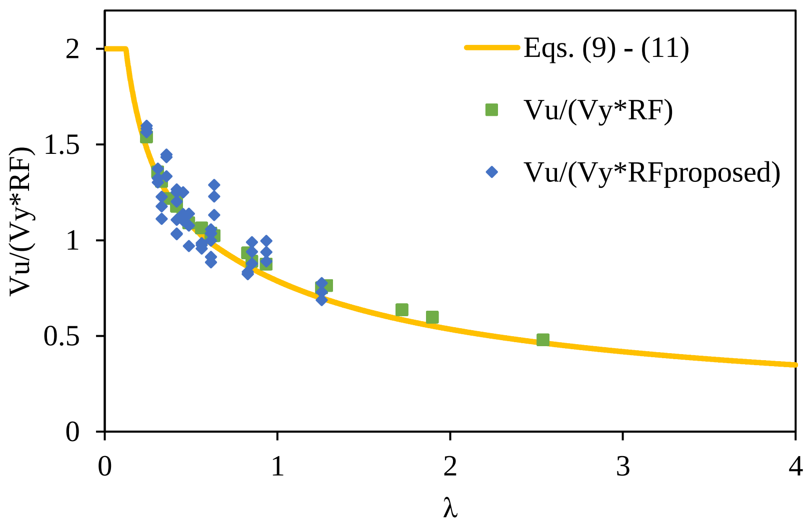


Figure 6: Distribution of FE shear capacities of LCBs after applying the reduction factor and the current DSM shear design curve

5. Conclusions

In this paper the effect of reduced support restraints on the shear behaviour of stainless steel LCBs was investigated by means of FE modelling. Initially, FE models were validated with available full-height WSP test results from the literature and then parametric FE studies were conducted. Those results were then utilised to propose a reduction factor for DSM design rules to account for shear capacity loss due to the reduced support restraints. It can be concluded that shear capacity of stainless steel LCBs reduces with the reduction of the support height, where about 50 % reduction in shear capacity can be observed for 60 % reduction in WSP height while this capacity reduction is higher in compact sections compared to slender sections. In order to account for these capacity reductions, a reduction factor for DSM shear design rules were proposed based on the FE results. Proposals found to be agree well with the FE results where actual-to-predicted shear capacity reduction ratio for the proposal has a mean and COV of 1.00 and 0.097, respectively. However, the applicability of this reduction factor is limited to compact sections. Therefore, further investigations are underway to address this limitation.

References

American Iron and Steel Institute (2007) *AISI–S100–2007. North American specification for the design of cold-formed steel structural members.* Washington (DC), AISI.

Arrayago, I., Real, E. & Gardner, L. (2015) Description of stress-strain curves for stainless steel alloys. *Materials and Design*. 87 540–552.

Dissanayake, D. M. M. P., Poologanathan, K., Gunalan, S., Tsavdaridis, K. D., Nagaratnam, B. & Wanniarachchi, K. S. (2019a) Numerical modelling and shear design rules of stainless steel lipped channel sections. *Journal of Constructional Steel Research (In press).*

Dissanayake, M., Poologanathan, K., Gunalan, S., Tsavdaridis, K. D. & Nagaratnam, B. (2019b) Finite element analyses of cold-formed stainless steel beams subject to shear. *Special Issue: Proceedings of Nordic Steel 2019.* 3 (3-4) 931–936.

Dissanayake, D. M. M. P., Poologanathan, K., Gunalan, S., Tsavdaridis, K. D. & Degtyareva, N. (2019c) Effect of corner strength enhancement on shear behaviour of stainless steel lipped channel sections. Submitted to *The 10th International Conference on Structural Engineering and Construction Management, 12–14 December 2019, Kandy, Sri Lanka.*

Fareed, I., Somadasa, W., Poologanathan, K., Gunalan, S., Corradi, M. & Sivapalan, S. (2019) Finite element analyses of cold-formed stainless steel beam with web openings in shear. *Special Issue: Proceedings of Nordic Steel 2019.* 3 (3–4) 907–912.

Gardner, L. & Nethercot, D. A. (2004) Numerical modelling of stainless steel structural components – A consistent approach. *Journal of Structural Engineering*. 130 (10) 1586–1601.

Keerthan, P. & Mahendran, M. (2011) Numerical modeling of litesteel beams subject to shear. *Journal of Structural Engineering*. 137(12) 1428-1439.

Keerthan, P. & Mahendran, M. (2012) Shear behaviour and strength of LiteSteel beams with web openings. *Advances in Structural Engineering.* 15(2) 171-184.

Keerthan, P. & Mahendran, M. (2013a) Experimental studies of the shear behaviour and strength of lipped channel beams with web openings. *Thin-Walled Structures.* 73 131-144.

Keerthan, P. & Mahendran, M. (2013b) Shear buckling characteristics of cold-formed steel channel beams. *International Journal of Steel Structures.* 13(3) 385-399.

Keerthan, P. & Mahendran, M. (2015a) Experimental investigation and design of lipped channel beams in shear. *Thin-Walled Structures.* 86 174–184.

Keerthan, P. & Mahendran, M. (2015b) Improved shear design rules of cold-formed steel beams. *Engineering Structures*. 99 603–615.

Keerthan, P., Mahendran, M. & Hughes, D. (2014) Numerical studies and design of hollow flange channel beams subject to combined bending and shear actions. *Engineering Structures*. 75 197-212.

Keerthan, P., Mahendran, M. & Narsey, A. (2015) Shear tests of hollow flange channel beams with real support conditions. *Structures.* 3 109–119.

Mahendran, M. & Keerthan, P. (2013) Experimental studies of the shear behavior and strength of LiteSteel beams with stiffened web openings. *Engineering Structures*. 49 840-854.

Pham, C. H. & Hancock, G. J. (2012) Tension field action for cold-formed sections in shear. *Journal of Constructional Steel Research.* 72 168–178.

Pham, C. H., Zelenkin, D. & Hancock, G. J. (2017) Effect of flange restraints on shear tension field action in cold-formed C-sections. *Journal of Constructional Steel Research.* 129 42–53.

Standards Australia/Standards New Zealand (2005) *AS/NZS 4600 cold-formed steel structures.* Sydney, SA.

Sundararajah, L., Mahendran, M. & Keerthan, P. (2017a) New design rules for lipped channel beams subject to web crippling under two-flange load cases. *Thin-Walled Structures*. 119 421-437.

Sundararajah, L., Mahendran, M. & Keerthan, P. (2017b) Web crippling experiments of high strength lipped channel beams under one-flange loading. *Journal of Constructional Steel Research*. 138 851-866.